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Bored pile design in stiff clay I: codes of practice

1 **Paul J. Vardanega** MEngSc, MIEAust, A.M.ASCE
Graduate Researcher, Department of Engineering, University of Cambridge, UK

2 **Ekaterina Kolody** MSc
Geotechnical Engineer, Ove Arup and Partners, London, UK

3 **Stuart H. Pennington** BEng, BSurv, CEng, MICE
Geotechnical Engineer, Ove Arup and Partners, London, UK

4 **Paul R. J. Morrison** MSc, DIC, PhD, CEng, MIEI
Director, Ove Arup and Partners, London, UK

5 **Brian Simpson** OBE, FREng, MA, PhD, FICE
Fellow, Ove Arup and Partners, London, UK


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The assessment of the allowable bearing load of bored piles ‘floating’ in stiff clay is a standard engineering task. Although the soil mechanics is universal, engineers designing structures in different parts of the world will need to take into account the pertinent codes of practice. It will be helpful to compare such codes, especially in relation to their treatment of uncertainty in the design of bored piles. This paper presents a series of design calculations for a real set of geotechnical data using four international codes of practice: the Australian, American, European and Russian codes. The National Annexes of Ireland, the Netherlands and the UK are used in conjunction with the European code. This selection of countries covers the three Eurocode 7 design approaches (DA1, DA2 and DA3). A non-codified design method is used to provide a base case for comparative purposes with the six codified calculations. A companion paper investigates the issues of soil mechanics in pile design methods, uncertainty in soil parameters and settlement criteria.

Notation

A_b	area of pile base
c_u	undrained shear strength (kPa)
D	pile diameter (m)
E_d	design value of effect of actions
f_s	unit skin friction (kPa)
G	unfactored permanent load
I_L	liquidity index
I_P	plasticity index
K	testing benefit factor
L	length of pile in clay stratum (m)
N_c	bearing capacity factor
Q_b	base resistance of the pile (kN)
Q_d	design load of a bored pile (kN)
Q_s	shaft resistance of the pile (kN)
Q_T	geotechnical design resistance of a pile
Q_{work}	$G + V =$ working load (kN)
R_d	design value of resistance
$R_{d,g}$	design geotechnical strength
$R_{d,ug}$	design ultimate geotechnical strength
V	unfactored variable load
z	depth below top of clay stratum (m)
α	correlation factor between unit skin friction (f_s) and undrained shear strength (c_u)
β_1	partial factor on permanent load
β_2	partial factor on variable load
β_3	partial factor on c_u along pile shaft
β_4	partial factor on c_u at pile base
β_5	partial factor on pile shaft resistance
β_6	partial factor on pile base resistance
β_7	partial factor on design resistance
γ_{RD}	model factor used in the UK National Annex to Eurocode 7
ϕ_g	geotechnical reduction factor
ϕ_{gb}	basic geotechnical reduction factor
ϕ_{tf}	intrinsic test factor

1. Introduction

The design of piles in stiff over-consolidated clay is common in geotechnical engineering. The engineer uses judgement, experience, available site data and knowledge of soil mechanics to complete the design task and ensure designs are compliant with the code of practice in force in the relevant jurisdiction. In this paper the requirements of AS2159-2009 in Australia (Standards Australia, 2009), Eurocode 7 (BSI, 2010) in the European Union, the American Association of State Highway and Transportation Officials (AASHTO) load and resistance factor design (LRFD) bridge design specifications (AASHTO, 2007) in the USA and SNiP 2.02.03-85 (SNiP, 1985a) in Russia, are considered together with a simple lump factor of safety design method. The Eurocode

7 calculations are performed using three national annexes to show the effect of using each of the three design approaches in Eurocode 7. A key aim of the paper is to explore the different approaches to uncertainty and safety intrinsic in these codes, so that engineers may be better informed on how to achieve their customary safety standards when working with an unfamiliar code.

The design example in this paper is a single pile in stiff over-consolidated clay. The data are taken from a site in London. However, this review could equally be applied to other stiff over-consolidated clays such as the Keswick-Hindmarsh Clay of Adelaide, Old Bay Clay of San Francisco, Boom Clay of the Netherlands, Palaeogene Clay of Denmark or Voskrensky Clay of Moscow and so on. This problem can be tackled with varying degrees of rigour depending on the nature of the design project being completed and the design assumptions required. Only the collapse/ultimate limit state will be considered in this paper. Settlement/serviceability limit state considerations are examined in a companion paper.

2. Method of calculation

The design for 'collapse' or 'ultimate' limit states is based on undrained shear strength. Calculations based on effective stress parameters are considered in the companion paper. The 'α method' for pile design is used to calculate unit skin friction (f_s)

$$f_s = \alpha c_u \tag{1}$$

where f_s is the unit skin friction on the pile shaft and α is an empirical adhesion co-efficient linking undrained shear strength to f_s. A common assumption of $\alpha = 0{\cdot}5$ was adopted for the calculations (e.g. Meyerhof (1976) and Tomlinson (1986) suggested 0·45 after Skempton (1959)). Patel (1992) suggested that in London Clay $\alpha = 0{\cdot}6$ (for constant rate of penetration (CRP) tests) was reasonable. Some codes of practice mandate values of α but in London large amounts of available test data mean that $\alpha = 0{\cdot}5$ is commonly used and not unduly optimistic. c_u is the undrained shear strength of the clay (kPa).

For a clay deposit with a c_u value dependent on depth (z) the pile shaft resistance is calculated using Equation 2

$$Q_s = \pi D \alpha \int_0^L c_u \mathrm{d}z \tag{2}$$

where D is pile diameter (m); c_u is undrained shear strength (kPa); α is an empirical adhesion co-efficient; L is the length of pile in the clay stratum (m); and z is depth of clay stratum (m).

The base capacity in clays is generally determined using

$$Q_b = A_b N_c c_u \tag{3}$$

where A_b is the area of the base (m^2); N_c is the bearing capacity factor, which varies depending on the sensitivity and deformation characteristics of the clay, but is generally taken as 9 (e.g. Meyerhof, 1976); and c_u is the undrained shear strength (kPa) at the base.

The geotechnical resistance (Q_T) of a pile is determined using the following equation

$$Q_T = Q_s + Q_b \tag{4}$$

3. General design formula

Partial factors can be applied at various stages in the calculation process. In limit state design these reflect the different sources of uncertainty. Equation 5 shows the general pile design formula with partial factors, denoted as β_1 to β_7. In the codes of practice reviewed, various combinations of partial factors are used. No one approach utilises all the possible partial factors shown below. Therefore, some will be given a value of unity when the design approach does not specify a value for them. The 'β' factors shown in Equation 5 all take a value greater than or equal to unity. Equation 5 could be re-written to make the factors less than unity by changing them from multipliers to divisors or vice-versa. As an example, using only one factor (β_7) with a non-unity value would represent a design approach with a single overall factor of safety.

Since different codes use different terminologies and symbols for various quantities the 'generic' notation defined in Equation 5 will be used so that the different approaches can be easily compared. In this paper, the terminology of most recent codes will be adopted, in which the 'design value' of a parameter is one that incorporates margins or factors of safety. For an economic design the design load Q_d equals the design resistance. That is

$$\begin{aligned} Q_d &= \beta_1 G + \beta_2 V \\ &= \left[\frac{\pi D \alpha \int_o^L (c_u/\beta_3) \mathrm{d}z}{\beta_5} + \frac{A_b N_c (c_u/\beta_4)}{\beta_6} \right] \Big/ \beta_7 \end{aligned} \tag{5}$$

where G is the unfactored permanent load; V is the unfactored variable load; β_1 is the partial factor on permanent load (G); β_2 is the partial factor on variable load (V); β_3 is the partial factor on c_u along pile shaft; β_4 is the partial factor on c_u at pile base; β_5 is the partial factor on pile shaft resistance; β_6 is the partial factor on pile base resistance; and β_7 is the partial factor on combined shaft and base resistance

The term 'partial factor' is used for the 'β' terms to include all types of factors used in the various codes (factor of safety, partial factor, model factor and so on).

In order to compare the different codes fairly a quantity Q_{work}, termed the working load, is defined

6. $$Q_{\text{work}} = G + V$$

The value of Q_{work} includes no partial factors and G and V are unfactored loads.

4. Design problem and site data

To illustrate how independently developed codes of practice affect the design of a single pile in clay, as well as the influence that different methods of analysis have on the resulting design, the following example is presented.

An engineer has been asked to determine the allowable working load (Q_{work}, defined as the combined unfactored permanent plus variable load) of the piles shown in Figure 1. In this paper, for simplicity, eccentricity of loading is not considered. The pile to be designed is a bored, straight-shafted, cast-in-place concrete pile, with no load testing carried out on the site. The variable load (V) is assumed to be 0·25 times the permanent load (G). This is a generic permanent to variable load ratio that has been taken to simulate a standard structure. Information based on Simpson *et al.* (1980) has been used to provide ground investigation data for the London Clay deposit. Data were collected from six boreholes with locations as shown on Figure 2. The Atterberg limits are summarised in Figure 3. Data from 102 mm unconsolidated, undrained (UU) triaxial tests (Figure 4) and correlated Standard Penetration Test (SPT) data (Figure 5) show the variation of undrained shear strength (c_u) with depth in the clay.

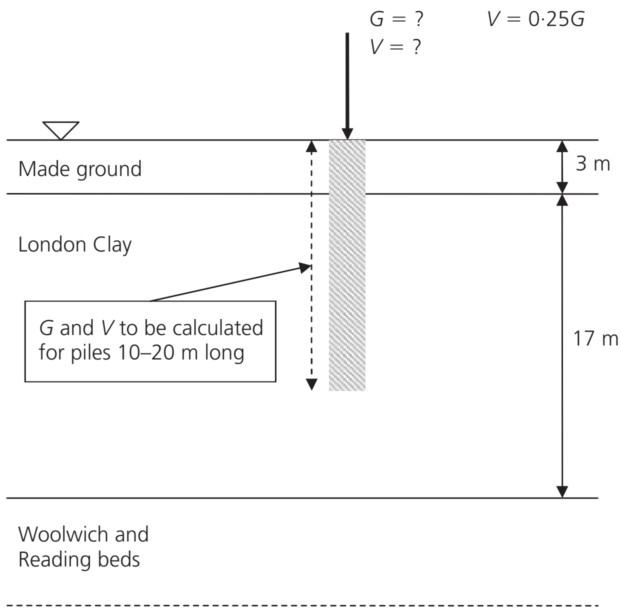

Figure 1. Idealised soil profile and pile to be designed

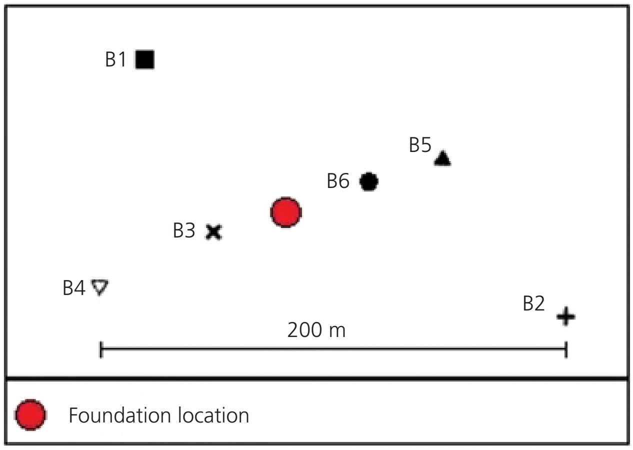

Figure 2. Location of foundation and boreholes (1 cm = 30 m)

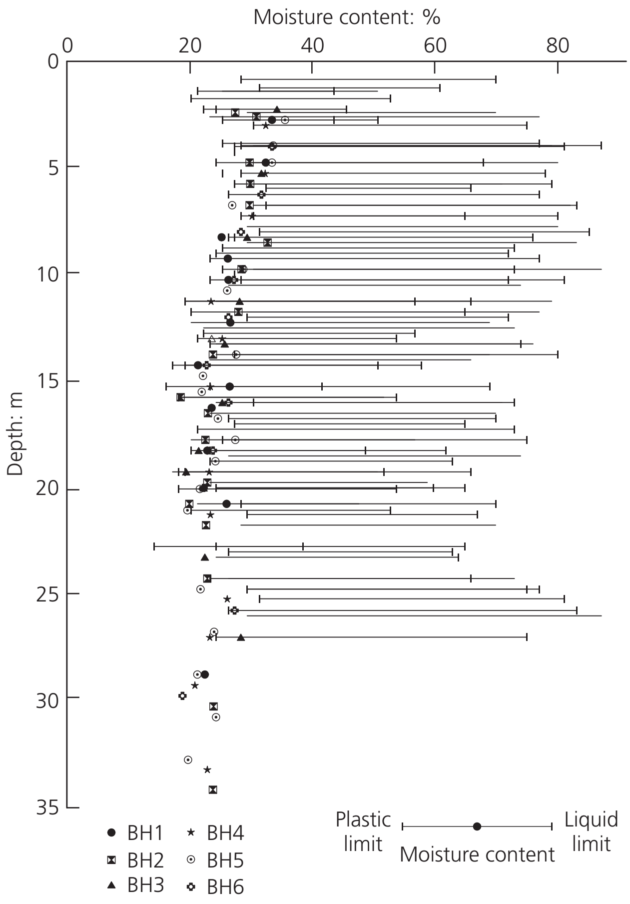

Figure 3. Site Atterberg limits

To convert the SPT N_{60} values to c_u, Equation 7 was used (see Figure 6)

$$c_u = 4{\cdot}4(N_{60}) \tag{7}$$

Plasticity index (I_p) varies on site from about 30% to 50% (Figure 3). Using the correlation from Figure 6 for $I_p = 30\%$ gives a multiplier on N_{60} for c_u of about 4·7 and for $I_p = 50\%$ a multiplier of 4·2. For an N_{60} equal to 40 the range in I_p values would correspond to a range of c_u from 168 kPa to 188 kPa as I_p decreases. An average I_p of 40% was adopted for the following analysis. Comments on Stroud (1974) with respect to the lack of statistical treatment have been made (Reid and Taylor, 2010). Vardanega and Bolton (2011) showed that a power curve, drawn through Stroud's data (Figure 6) is a good statistical relationship that could be fitted to the data. The coefficient of determination (R^2) of the regression line is 0·37 ($R^2 = 0{\cdot}37$). The regression curve is similar to Stroud's original line and the use of either curve results in Equation 7 for a plasticity index of 40%. There is a divergence between the two curves at low and high plasticity indices.

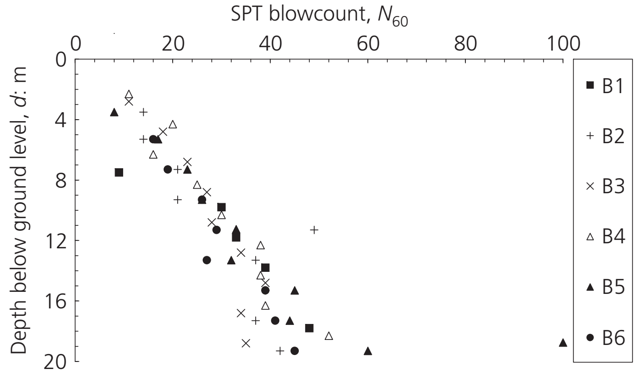

Figure 5. Results from SPT measurements

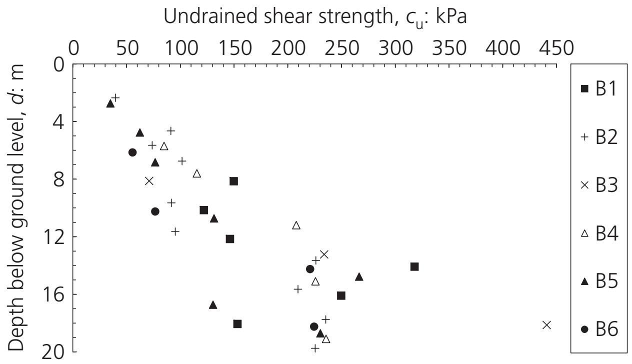

Figure 4. Results from triaxial tests

5. Undrained shear strength (c_u) relationship with depth

Figure 7 shows the *combined data* from Figures 4 and 5 (converted SPTs and data from 102 mm UU triaxial tests), with linear regression lines through the undrained strength data of individual boreholes. The slope does not vary considerably for the six boreholes. The data points are not highly scattered with

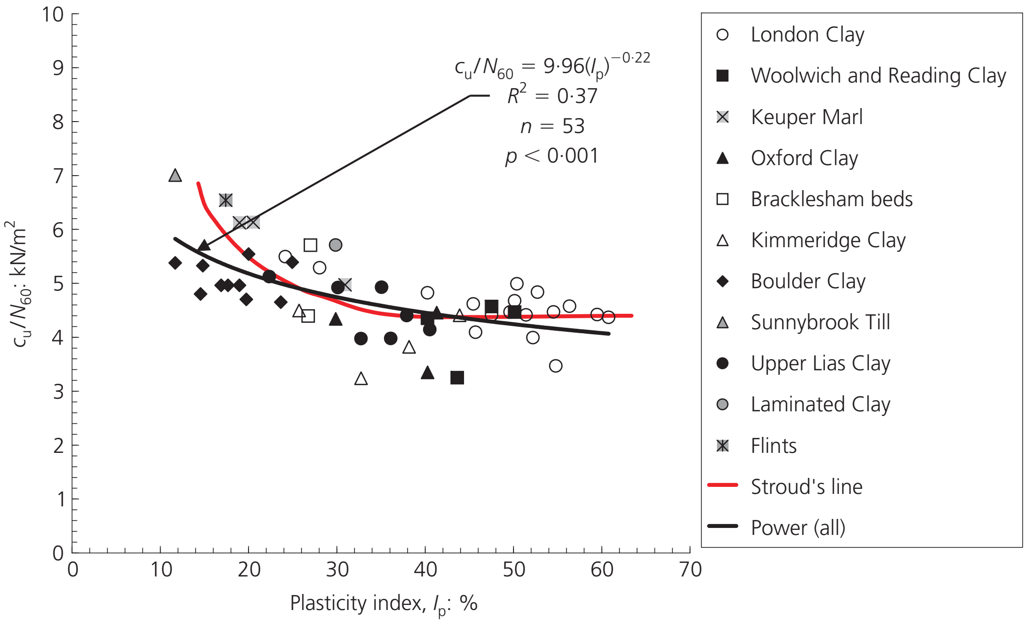

Figure 6. Relationship between c_u/N_{60} and I_p for a variety of clays (replotted from Stroud (1974) and Vardanega and Bolton (2011))

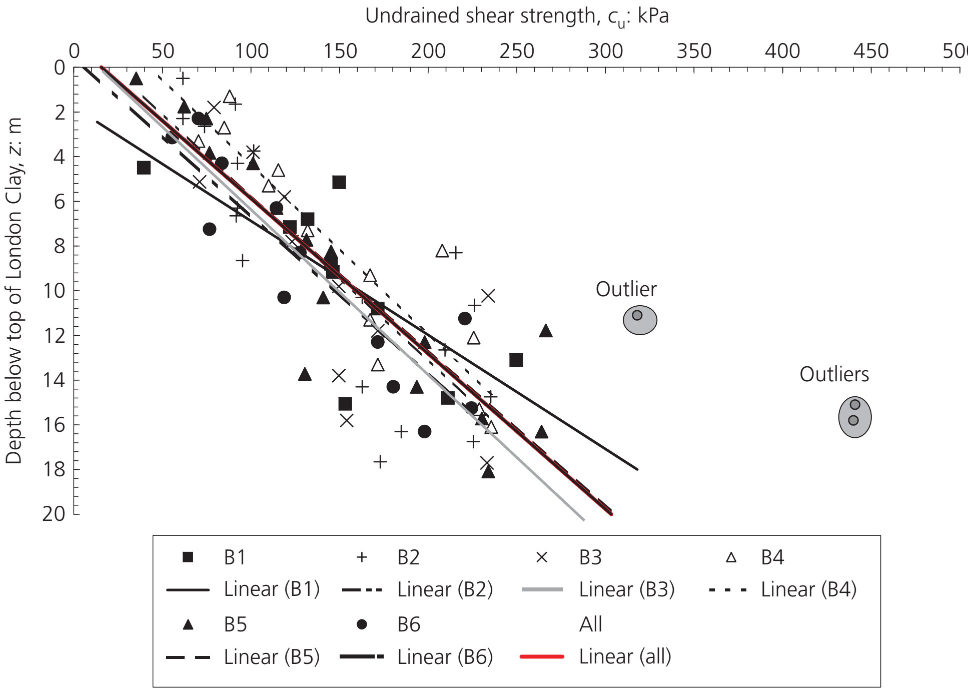

Figure 7. Linear regression lines to characterise c_u increase with depth (individual boreholes)

the R^2 for the individual lines varying from about 0·55 to 0·83. This is not an unexpected characteristic of the London Clay which was deposited in quiet, deep water conditions and has locally had the same geological history of overburden and erosion. Vertical variation is much more likely on this scale, as original deposition conditions change with depth. For instance, locations 50 m apart horizontally (deposited at the same time) may be much more similar than locations 1 m apart vertically (deposited many years apart). Of course, there could be some slight rotation of the bedding, but not very much, and there are occasional anomalies such as faults and pingos.

Regression lines in Table 1 are unsuitable as a design line if they imply negative or unreasonable shear strength at the top of the clay. In a stiff, overconsolidated deposit the mere ability for people to 'stand on the soil' implies some shear strength is present. This geological fact means that a blind regression is not advised for the determination of the design c_u profile. Indeed there is no geological reason for a straight line to be used. The reason for adopting a straight line is that a single gradient can be easily defined, thus simplifying the computation of skin friction.

Many engineers could offer a variety of possible design lines/relationships to characterise the data shown in Figure 7. In this paper, it is assumed that the characteristic value or 'cautious estimate' described by the Eurocode is given by Equation 8a. The 'representative value' used in conventional design is given by

Borehole	c_u relationship	R^2	n
B1	$c_u = 19{\cdot}61z - 34{\cdot}84$	0·545	10
B2	$c_u = 13{\cdot}52z + 22{\cdot}12$	0·679	19
B3	$c_u = 13{\cdot}73z + 15{\cdot}01$	0·603	11
B4	$c_u = 13{\cdot}09z + 43{\cdot}35$	0·831	13
B5	$c_u = 14{\cdot}43z + 16{\cdot}94$	0·809	16
B6	$c_u = 14{\cdot}03z + 6{\cdot}74$	0·815	12
All	$c_u = 14{\cdot}39z + 15{\cdot}50$	0·711	81

Table 1. Undrained shear strength relationships for each borehole (coefficient of determination, R^2, and number of data points, n, used in each regression shown)

Equation 8b and was derived by an eye fit to the data. Both lines are plotted on Figure 8. Equation 8a is drawn at the 25th percentile (of the total number of data points) parallel to the lower bound trace of the data (also shown of Figure 8). The lower bound trace is used to define the gradient of Equation 8a. This methodology for defining the gradient of the shear strength with depth works because there are no obvious outliers to the lower bound of the data set. The AASHTO and SNiP calculations make use of 'average value' soil parameters. In the AASHTO guide, clause 10.4.6.2.2 states 'correlations for c_u based on SPT tests should be avoided'. Therefore, for the AASHTO calculations only

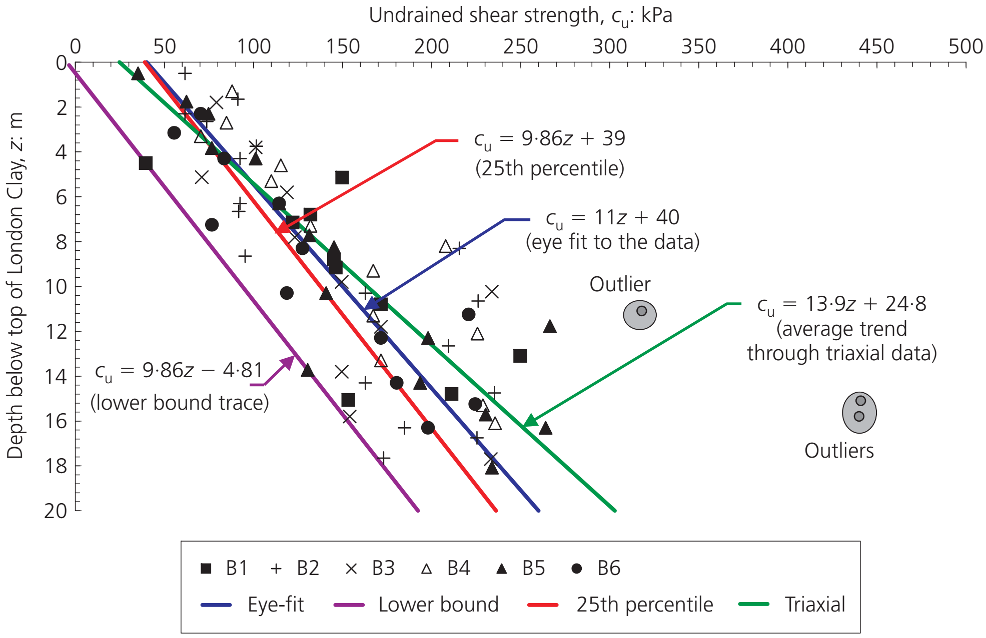

Figure 8. Relationships between c_u and depth used in this paper

the triaxial data were used to characterise the strength increase with depth; this is given by Equation 8c and the line shown on Figure 8. For SNiP, calculations are based on liquidity index and not c_u.

8a. $$c_u = 39 + 9{\cdot}86z \qquad \text{(kPa)}$$

8b. $$c_u = 40 + 11z \qquad \text{(kPa)}$$

8c. $$c_u = 24{\cdot}8 + 13{\cdot}9z \qquad \text{(kPa)}$$

The rationale for a line similar to Equation 8a is that it lies below the mean of the data and is a cautious estimate of strength and therefore a good choice for the 'characteristic value' that is required for determination of soil properties in Eurocode 7. It is acknowledged that a 5th percentile line could be used, but that this is an extremely conservative view of what is essentially a large amount of data (Simpson *et al.*, 2009).

6. Conventional design

For comparison with the codes considered in this paper a simple design method is presented as the base case. The design is based on a global factor of safety.

BS 8004 (clause 7.3.8) (BSI, 1986) states

> in general, an appropriate factor of safety for a single pile would be between two and three. Low values within this range may be applied where the ultimate bearing capacity has been determined by a sufficient number of loading tests or where they may be justified by local experience; higher values should be used when there is less certainty of the value of the ultimate bearing capacity.

(BS 8004 has now been superseded by BS EN 1997-1:2004).

For the purpose of this example a value of 3·0 is adopted herein assuming that no pile load testing is carried out. For 'conventional design calculations' Equation 5 reduces to Equation 9

5. $$Q_d = \beta_1 G + \beta_2 V = \left[\frac{\pi D\alpha\int_o^L (c_u/\beta_3)\mathrm{d}z}{\beta_5} + \frac{A_b N_c (c_u/\beta_4)}{\beta_6}\right] \Big/ \beta_7$$

9. $$Q_d = G + V = \left(\pi D\alpha\int_o^L c_u \mathrm{d}z + A_b N_c c_u\right) \Big/ \beta_7$$

where $\beta_7 = 3{\cdot}0$.

Calculations for a conventional design, for a 15 m long (12 m into the clay), 0·45 m diameter pile follows

10. $$Q_\mathrm{d} = G + V = \left(\pi D\alpha\int_o^L c_\mathrm{u} dz + A_\mathrm{b} N_\mathrm{c} c_\mathrm{u}\right) \Big/ 3{\cdot}0$$

11. $$\begin{aligned} Q_\mathrm{d} &= G + V \\ &= \left(\pi D(0{\cdot}5)\int_0^{12} (40 + 11z)\mathrm{d}z + A_\mathrm{b} N_\mathrm{c} c_\mathrm{u}\right) \Big/ 3{\cdot}0 \end{aligned}$$

($\alpha = 0{\cdot}5$ for bored piles in London).

12. $$Q_\mathrm{d} = G + V = (899{\cdot}1 + 246{\cdot}2)/3{\cdot}0$$

13. $$Q_\mathrm{d} = G + V = 381{\cdot}8 \ \ \mathrm{kN}$$

In this case $Q_\mathrm{d} = Q_\mathrm{work}$ as no factors are applied to the loads.

The split between G and V based on V being 25% of G returns values of
$G = 305{\cdot}4$ kN
$V = 76{\cdot}4$ kN
$Q_\mathrm{work} = 381{\cdot}8$ kN.

7. AS2159-2009 (Australia)

The Australian approach to designing piles makes use of partial factors with loads being factored separately from the capacities. A single factor is applied to the calculated geotechnical resistance, termed the 'geotechnical reduction factor', applied to the calculated resistances, not the soil parameters.

AS2159-2009 (Standards Australia, 2009) directs the engineer to AS/NZS 1170.0 (Standards Australia, 2002) (structural design actions) for the load factors. The two relevant combinations for a pile are most likely to be the greater of: $1{\cdot}2G + 1{\cdot}5V$ or $1{\cdot}35G$. Since, for this design, $V/G = 0{\cdot}25$, the critical case is $1{\cdot}2G + 1{\cdot}5V$. Since this paper is only considering collapse limit states, serviceability and actions induced by ground movements are not considered. Earthquake loading is also not considered.

Clause 4.3.1 of AS2159-2009 states that the design geotechnical strength ($R_\mathrm{d,g}$) must not be less than the design action effect (E_d). For a single pile, not a group, E_d will be taken as the load imparted from the pile cap to the individual pile. The code defines $R_\mathrm{d,g}$

14. $$R_\mathrm{d,g} = \phi_\mathrm{g} R_\mathrm{d,ug}$$

$R_\mathrm{d,ug}$ is the design ultimate geotechnical strength, determined from site data and calculation methods; ϕ_g is the geotechnical reduction factor (not to be confused with friction angle)

15. $$\phi_\mathrm{g} = \phi_\mathrm{gb} + (\phi_\mathrm{tf} - \phi_\mathrm{gb})K \geqslant \phi_\mathrm{gb}$$

ϕ_tf is the intrinsic test factor; K is the testing benefit factor; and ϕ_gb is the basic geotechnical reduction factor.

In this example no load testing is being considered so $\phi_\mathrm{g} = \phi_\mathrm{gb}$ as calculated in the next section. There is a testing benefit factor (K) in the Australian code which allows ϕ_g to be reduced if load testing is performed. K is determined using the percentage of piles statically or dynamically tested (see clause 4.3.1 of AS2159-2009).

7.1 Determination of basic geotechnical reduction factor

To determine the basic geotechnical reduction factors the individual risk ratings (IRRs) (Table 2) are assigned to each of the risk factors listed in Table 3. This approach to determine geotechnical reduction factors was explained in Poulos (2004).

ϕ_gb is determined from the average risk rating (ARR), calculated using Equation 16, and then using Table 4. Design of a single pile, not in a large group, is treated as a design with *low redundancy.*

16. $$\mathrm{ARR} = \Sigma(w_i \mathrm{IRR}_i)/\Sigma w_i$$

16a. $$\mathrm{ARR} = 36{\cdot}5/14{\cdot}5 = 2{\cdot}52$$

16b. $$\phi_\mathrm{gb} = 0{\cdot}52 \qquad \text{(low to moderate risk)}$$

The Australian method gives more responsibility to the engineer

Risk level	Very low	Low	Moderate	High	Very high
Individual risk rating	1	2	3	4	5

Table 2. Individual risk rating (after T4.3.2(B) AS2159)

Risk factor	Weighting factor, w_i	Individual risk rating (IRR)	Comments
Site			
Geological complexity of site	2	2	Well-understood soil strata, London Clay is widely studied and lots of testing done on this site Low risk
Extent of ground investigation	2	2	Relatively deep boreholes with lots of test data down to pile depth proposed Low risk
Amount and quality of geotechnical data	2	3	Undrained triaxial data and SPTs taken Moderate risk
Design			
Experience with similar foundations in similar geological conditions	1	2	Bored piles in London Clay, very common Low risk
Method of assessment of geotechnical parameters for design	2	3	Combination on conventional laboratory triaxial testing and well-established site correlations on SPT data Moderate risk
Design method adopted	1	3	Simple empirical methods are being employed here but both are well calibrated for London Clay Moderate risk
Method of utilising results of in situ test data and installation data	2	2	Using the 25th percentile of the data to determine c_u against depth relationship Low risk
Installation			
Level of construction control	2	3	Since only performing a desktop study, conventional construction processes will be used, limited degree of professional involvement. Moderate risk
Level of monitoring	0·5	3	Assume little long-term monitoring as this is a simple project Moderate risk

Note: The pile design shall include the risk circumstances for each individual risk category and consideration of all of the relevant site and construction factors (AS2159 T4.3.2(A)).

Table 3. Geotechnical risk factors, weightings and ratings

to determine the reduction factor on the geotechnical calculations. It bounds the value of ϕ_{gb} between 0·67 and 0·4 for low-redundancy systems and between 0·76 and 0·47 for high-redundancy systems. For low-redundancy problems, this is akin to dividing the calculated resistances by 1·50 for very low risk and 2·5 for very high risk, as shown in the 'Equivalent β_7' column in Table 4; that is, the 'partial factor' on the geotechnical resistance is between 1·5 and 2·5 with the loading being considered separately.

7.2 Design calculations

For design to AS2159-2009 Equation 5 reduces to Equation 17

$$Q_d = \beta_1 G + \beta_2 V = \left[\frac{\pi D \alpha \int_o^L (c_u/\beta_3)\mathrm{d}z}{\beta_5} + \frac{A_b N_c (c_u/\beta_4)}{\beta_6}\right] \Big/ \beta_7 \quad (5)$$

$$Q_d = \beta_1 G + \beta_2 V = \left(\pi D \alpha \int_o^L c_u \mathrm{d}z + A_b N_c c_u\right) \Big/ \beta_7 \quad (17)$$

Range of average risk rating (ARR)	Overall risk category	Low-redundancy systems		High-redundancy systems	
		ϕ_{gb}	Equivalent β_7	ϕ_{gb}	Equivalent β_7
ARR ⩽ 1·5	Very low	0·67	1·50	0·76	1·32
1·5 < ARR ⩽ 2·0	Very low to low	0·61	1·64	0·70	1·43
2·0 < ARR ⩽ 2·5	Low	0·56	1·79	0·64	1·56
2·5 < ARR ⩽ 3·0	Low to moderate	0·52	1·92	0·60	1·67
3·0 < ARR ⩽ 3·5	Moderate	0·48	2·08	0·56	1·79
3·5 < ARR ⩽ 4·0	Moderate to high	0·45	2·22	0·53	1·89
4·0 < ARR ⩽ 4·5	High	0·42	2·38	0·50	2·00
ARR > 4·5	Very high	0·40	2·50	0·47	2·13

Table 4. Basic geotechnical strength reduction factor for average risk rating

where
β_1, partial factor on permanent load =1·2 (AS1170)
β_2, partial factor on variable load = 1·5 (AS1170)
$\beta_7 = 1/\phi_g = 1{\cdot}92$.

For a 15 m long (12 m into the clay), 0·45 m diameter pile

$$Q_d = \beta_1 G + \beta_2 V = \left[\pi D\alpha \int_o^{12} (9{\cdot}86z + 39)\mathrm{d}z + A_b N_c (9{\cdot}86 \times 12 + 39)\right] \Big/ \beta_7 \quad (18)$$

$$Q_d = 1{\cdot}2G + 1{\cdot}5V = (832{\cdot}6 + 225{\cdot}2)/1{\cdot}92 \quad (19)$$

Taking
$V = 0{\cdot}25G$
$G = 349{\cdot}8$ kN
$V = 87{\cdot}5$ kN
$Q_{work} = 437{\cdot}3$ kN.
The equivalent overall FOS = (832·6 + 225·2)/437·3 = 2·42

8. Introduction to Eurocode 7

Eurocode 7 (EN 1997-1:2004 (BSI, 2010)) is a limit state code which employs partial factors. After checking the relevant limit states, the designer must ensure that the design value of the effect of actions, E_d, (the design loads) is less than or equal to the design value of the resistance, R_d (the design capacity)

$$E_d \leqslant R_d \quad (20)$$

For design by calculation, Eurocode 7 presents three design approaches: DA1, DA2 and DA3. Partial factors can be applied to the actions 'A' (i.e. the loads), the material properties 'M' (e.g. soil strengths) and the resistances 'R' (e.g. skin friction). Different design approaches use different combinations of partial factors. In order for the code to be used within a particular country, the national standards body of that country is required to produce a national annex (NA). The NA will specify which design approach(es) is/are permitted for construction in that country, and specifies the values of the partial factors to be used. In order to demonstrate the use of each design approach, three countries have been selected on the basis of their NA choice: the UK for DA1, Ireland for DA2 and the Netherlands for DA3.

9. EC 7 – design approach 1 (UK national approach)

9.1 Partial factors

This design approach is the one adopted by the UK NA to Eurocode 7 (BSI, 2007). In this design approach two sets of calculations are performed (DA1-1 and DA1-2), with the partial factors shown in Tables 5 and 6.

9.2 Model factor

Paragraph 2.4.1(8) of Eurocode 7 states: 'If needed, a modification of the results from the model may be used to ensure that the design calculation is either accurate or errs on the side of safety.' Paragraph 2.4.1 (9) states

Description	Partial factor	β term
Variable load	1·5	β_2
Permanent load	1·35	β_1
Skin friction	1·0	β_5
Base resistance	1·0	β_6

Table 5. DA1-1 partial factors used

Description	Partial factor	β term
Variable load	1·3	β_2
Permanent load	1·0	β_1
Skin friction	1·5 (driven piles)	β_5
	1·6 (bored piles)	
Base resistance	1·7 (driven piles)	β_6
	2·0 (bored piles)	

Note: Partial factors on resistances can be reduced with explicit verification of serviceability limit state (not applicable for this example).

Table 6. DA1-2 partial factors used

> if the modification of the results makes use of a model factor, it should take account of: the range of uncertainty in the results of the method of analysis; any systematic errors known to be associated with the method of analysis.

The UK NA introduces a model factor termed γ_{Rd}. In this example it is applied to the calculated shaft and base resistances to account for the fact that the analysis model is empirically based. The UK NA requires a value of 1·4 (which would be reduced to 1·2 if there were load testing). This term is represented in Equation 6 at the β_7 term; for more information on pile design to Eurocode 7 see Bond and Simpson (2010).

9.3 Design calculations

For a DA1-1 calculation Equation 5 reduces to Equation 25 and for a DA1-2 calculation Equation 5 reduces to Equation 22, assuming that no load testing is carried out

$$Q_{\mathrm{d}} = \beta_1 G + \beta_2 V = \left[\frac{\pi D\alpha \int_o^L (c_{\mathrm{u}}/\beta_3)\mathrm{d}z}{\beta_5} + \frac{A_{\mathrm{b}} N_{\mathrm{c}} (c_{\mathrm{u}}/\beta_4)}{\beta_6}\right] \Big/ \beta_7 \tag{5}$$

DA1-1; terms β_3, β_4, β_5 and β_6 are equal to unity and have been omitted

$$Q_{\mathrm{d}} = 1{\cdot}35G + 1{\cdot}5V = \left[\pi D\alpha \int_o^L c_{\mathrm{u}}\mathrm{d}z + A_{\mathrm{b}} N_{\mathrm{c}} c_{\mathrm{u}}\right] \Big/ 1{\cdot}4 \tag{21}$$

DA1-2; terms β_1, β_3 and β_4 are equal to unity and have been omitted

$$Q_{\mathrm{d}} = G + 1{\cdot}3V = \left[\frac{\pi D\alpha \int_o^L c_{\mathrm{u}}\mathrm{d}z}{1{\cdot}6} + \frac{A_{\mathrm{b}} N_{\mathrm{c}} c_{\mathrm{u}}}{2{\cdot}0}\right] \Big/ 1{\cdot}4 \tag{22}$$

For the 15 m pile (12 m into the clay) of 0·45 m diameter
DA1-1

$$Q_{\mathrm{d}} = 1{\cdot}35G + 1{\cdot}5(0{\cdot}25G) = \left[\frac{832{\cdot}6}{1{\cdot}0} + \frac{225{\cdot}2}{1{\cdot}0}\right] \Big/ 1{\cdot}4 \tag{23}$$

$$1{\cdot}725G = [832{\cdot}6 + 225{\cdot}2]/1{\cdot}4 \tag{24}$$

$G = 438{\cdot}0$ kN
$V = 109{\cdot}5$ kN
$Q_{\mathrm{work}} = 547{\cdot}5$ kN
The equivalent factor of safety is 1057·8/547·5 = 1·93.
DA1-2 (governs)

$$Q_{\mathrm{d}} = 1{\cdot}0G + 1{\cdot}3(0{\cdot}25G) = \left[\frac{832{\cdot}6}{1{\cdot}6} + \frac{225{\cdot}2}{2{\cdot}0}\right] \Big/ 1{\cdot}4 \tag{25}$$

$$1{\cdot}325G = (832{\cdot}6/1{\cdot}6 + 225{\cdot}2/2)/1{\cdot}4 \tag{26}$$

$G = 341{\cdot}2$ kN
$V = 85{\cdot}3$ kN
$Q_{\mathrm{work}} = 426{\cdot}5$ kN
The equivalent factor of safety is 1057·8/426·5 = 2·48.

10. EC 7 – design approach 2 (Irish national annex)

To demonstrate the use of DA2 for the calculation of pile load carrying capacity, the Irish NA (NSAI, 2005) has been selected. The Irish NA is unique in that it allows for any of the three design approaches to be used for geotechnical works.

10.1 Design parameters

Table 7 presents the parameters to be used for the Irish adoption of DA2.

10.2 Design calculation

Therefore, for DA2 design to the Irish NA Equation 5 reduces to Equation 27

Description	Partial factor	β term
Variable load	1·35	β_2
Permanent load	1·5	β_1
Skin friction	1·1	β_5
Base resistance	1·1	β_6

Table 7. DA2 parameters

5. $$Q_d = \beta_1 G + \beta_2 V = \left[\frac{\pi D \alpha \int_o^L (c_u/\beta_3) dz}{\beta_5} + \frac{A_b N_c (c_u/\beta_4)}{\beta_6} \right] \Big/ \beta_7$$

27. $$Q_d = \beta_1 G + \beta_2 V = \left(\frac{\pi D \alpha \int_o^L c_u dz}{\beta_5} + \frac{A_b N_c c_u}{\beta_6} \right) \Big/ \beta_7$$

NA.2.19 in the Irish NA requires a model factor of 1·75 to be applied to the shaft and base resistance factors for pile design.

28. $$Q_d = 1{\cdot}35G + 1{\cdot}5V = \left(\frac{\pi D \times 0{\cdot}5 \int_o^L c_u dz}{1{\cdot}1} + \frac{A_b N_c c_u}{1{\cdot}1} \right) \Big/ 1{\cdot}75$$

By way of example, for a 15 m long (12 m into the stiff clay), 0·45 m diameter pile with V assumed to be 0·25G this reduces to

29. $$Q_d = 1{\cdot}35G + 1{\cdot}5(0{\cdot}25G) = \left(\frac{832{\cdot}6}{1{\cdot}1} + \frac{225{\cdot}2}{1{\cdot}1} \right) \Big/ 1{\cdot}75$$

$G = 318{\cdot}6$ kN
$V = 79{\cdot}6$ kN
$Q_{work} = 398{\cdot}2$ kN
The equivalent FOS = 1057·8/398·2 = 2·66

11. EC 7 – design approach 3 (Netherlands national annex)

The Dutch use design approach 3 (NEN, 2007). The partial factors are applied to the soil strength and the base and shaft resistances (β_5 and β_6).

5. $$Q_d = \beta_1 G + \beta_2 V = \left[\frac{\pi D \alpha \int_o^L (c_u/\beta_3) dz}{\beta_5} + \frac{A_b N_c (c_u/\beta_4)}{\beta_6} \right] \Big/ \beta_7$$

30. $$Q_d = 1{\cdot}0G + 1{\cdot}0V = \left[\frac{\pi D \alpha \int_o^L (c_u/1{\cdot}35) dz}{1{\cdot}8} + \frac{A_b N_c (c_u/1{\cdot}35)}{1{\cdot}8} \right]$$

11.1 Design calculation

For a 15 m long pile (12 m into the clay) with $\alpha = 0{\cdot}5$ and 0·45 m diameter, where $V = 0{\cdot}25G$

31. $$Q_d = G + (0{\cdot}25G) = \left(\frac{832{\cdot}6/1{\cdot}35}{1{\cdot}8} + \frac{225{\cdot}2/1{\cdot}35}{1{\cdot}8} \right)$$

$G = 348{\cdot}2$ kN
$V = 87{\cdot}1$ kN
$Q_{work} = 435{\cdot}3$ kN
The equivalent FOS = (1057·8/435·3) = 2·43

12. American Association of State Highway and Transportation Officials (AASHTO)

The AASHTO bridge design specification (4th edition, AASHTO, 2007) specification adopts a limit state approach known in the USA as LRFD. This can be represented as follows

32. $$\sum \eta_i \gamma_i Q_i \leqslant \varphi_{qp} R_p + \varphi_{qs} R_s$$

The c_u relation with depth used in the AASHTO method is a mean value of triaxial data only, Equation 8c. Table 8 defines the parameters used in the AASHTO method and compares the notation used in AASHTO with that in the present paper.

For the design calculation according to AASHTO, Equation 5 reduces to Equation 33

5. $$Q_d = \beta_1 G + \beta_2 V = \left[\frac{\pi D \alpha \int_o^L (c_u/\beta_3) dz}{\beta_5} + \frac{A_b N_c (c_u/\beta_4)}{\beta_6} \right] \Big/ \beta_7$$

AASHTO notation	Notation in current paper	Description	Value
Q_{live}	V	Variable load	To be calculated
$Q_{permanent}$	G	Permanent load	To be calculated
η_i	η_i	Reliability factor	1·0
γ_i	β_1	Factor on permanent load	1·25
γ_i	β_2	Factor on variable load	1·75
ϕ_{qp}	$1/\beta_6$	Reduction factor on base resistance	0·4
ϕ_{qs}	$1/\beta_5$	Reduction factor on shaft resistance	0·45
c_u	c_u	Undrained shear strength	13·9z + 24·8 (see Figure 8)
Z	z	Depth	Varies
D	D	Pile diameter	Varies
—	$1/\beta_7$	Reduction factor on resistance	0·8
α	α	Adhesion factor	0·55 for $c_u/p_a \leqslant 1{\cdot}5$ $0{\cdot}55 - 0{\cdot}1(c_u/p_a - 1{\cdot}5)$ for $1{\cdot}5 \leqslant c_u/p_a \leqslant 2{\cdot}5$
N_c	N_c	Bearing capacity factor	9
R_s	Q_s	Shaft resistance	To be calculated
R_p	Q_b	Base resistance	To be calculated
A_s	A_s	Shaft area	To be calculated
A_p	A_b	Base area	To be calculated

Note 1: The value of η_i represents a conventional design, a conventional level of redundancy and a typical structure.
Note 2: The values for γ_i are for the Strength I load combination.
Note 3: The value of α is zero for the top 1·52 m (5 ft) and bottom one diameter.
Note 4: The value of $1/\beta_7$ applies to isolated piles.
Note 5: The value of $N_c = 6(1 + 0{\cdot}2(z/D)) \leqslant 9$

Table 8. AASHTO parameters

33. $$Q_d = \beta_1 G + \beta_2 V = \left(\frac{\pi D\alpha \int_o^L c_u \mathrm{d}z}{\beta_5} + \frac{A_b N_c c_u}{\beta_6} \right) \Big/ \beta_7$$

β_1, partial factor on permanent load; for AASHTO $= 1{\cdot}25$
β_2, partial factor on variable load; for AASHTO $= 1{\cdot}75$
β_5, partial factor on pile shaft resistance; for AASHTO $\beta_5 = 1/0{\cdot}45 = 2{\cdot}22$
β_6, partial factor on pile base resistance; for AASHTO $\beta_6 = 1/0{\cdot}40 = 2{\cdot}5$
β_7, partial factor on design resistance; for AASHTO $\beta_7 = 1/0{\cdot}8 = 1{\cdot}25$

34. $$Q_d = 1{\cdot}25G + 1{\cdot}75V = \left[\frac{\pi D\alpha \int_o^L (24{\cdot}8 + 13{\cdot}9z)\mathrm{d}z}{2{\cdot}22} + \frac{A_b N_c c_u}{2{\cdot}5} \right] \Big/ 1{\cdot}25$$

12.1 Design calculation

In the example calculation $\alpha = 0{\cdot}5$. AASHTO suggest a value of α that decreases with c_u/p_a which can be interpreted as an increase with depth. The value of α as suggested by AASHTO is used to compute the AASHTO capacities in the summary in Section 14, Figures 10 and 11.

For a 15 m long (12 m into the stiff clay), 0·45 m diameter pile

35. $$Q_d = 1{\cdot}6875G = \left(\frac{917{\cdot}8}{2{\cdot}22} + \frac{274{\cdot}3}{2{\cdot}5} \right) / 1{\cdot}25$$

$G = 248{\cdot}0$ kN
$V = 62{\cdot}0$ kN
$Q_{work} = 310{\cdot}0$ kN
The equivalent FOS = (1192·1/310) = 3·85.

If the shaft and base resistances calculated using the 25th percentile of soil data are compared with the factored capacities here then the equivalent FOS = (1057·8/310) =3·41).

13. SNiP (Russian approach)

The Russian design method for pile capacity is outlined in SNiP 2.02.03-85 (SNiP, 1985a). The method of determining bearing capacity is based on relating pile capacity (shaft and end bearing) to liquidity index (I_L) for fine-grained soils and to density and grain size for coarse-grained soils. The minimum liquidity index allowed in the SNiP is 0·2 for the skin resistance and 0·0 for the base resistance; these are higher than the site data would suggest (Figure 9), so use of these values will provide a lower bound result. Values for shaft adhesion as a function of liquidity index, taken from Table 2 of SNiP and values for base resistance as a function of liquidity index, taken from Table 7 of SNiP are shown as charts in Figures 13 and 14 in the Appendix.

Bearing capacity of a bored pile can be calculated using Equation 36, for which Table 9 gives a full explanation of the terminology

$$F_d = \gamma_c\left(\gamma_{cR} RA + u\gamma_{cf}\sum f_i h_i\right) \quad (36)$$

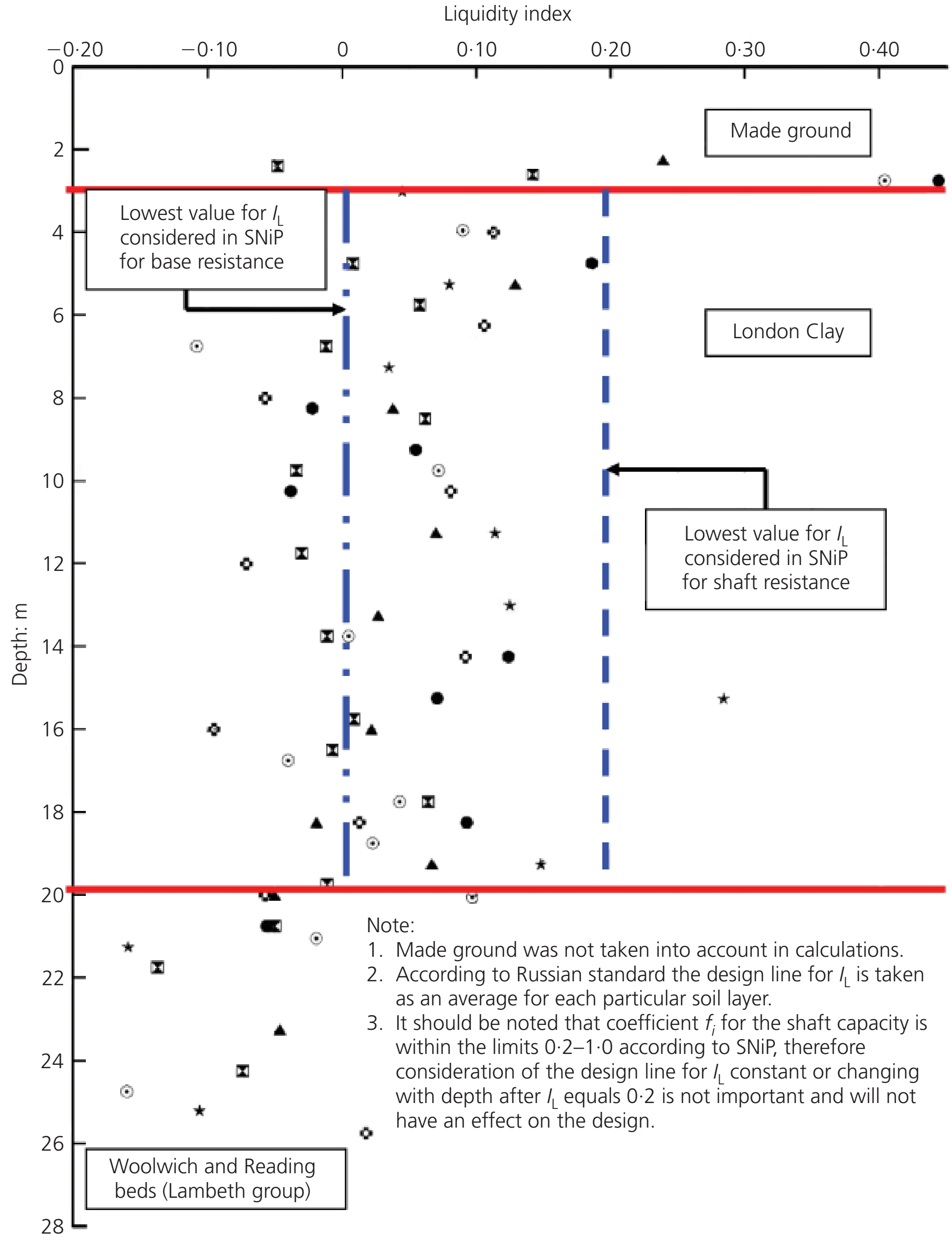

Figure 9. Design line through liquidity index data (SNiP calculation) (see Figure 3 for key)

SniP 2.02.03-85 notation	Equivalent notation in current paper	Notes
F_d	$Q_b + Q_s$	Bearing capacity, $F_d = \gamma_c(\gamma_{cR} RA + u\gamma_{cf}\sum f_i h_i)$
γ_c		Service factor for pile work in soil. If pile toe is in a soil with saturation degree of < 0·90 or in loose soils, then $\gamma_c = 0{\cdot}80$. In all other cases $\gamma_c = 1$. Taken as $\gamma_c = 1$ for the current paper
γ_{cR}		Service factor for soil beneath the bottom end of the pile; taken as for bored pile without under-reaming $\gamma_{cR} = 1$
R		Design resistance of soil at the pile tip level, kPa, accepted according to Table 7 SNiP 2·02·03-85, depending on liquidity index (I_L) and embedment depth of the base of the pile (design chart is shown in the Appendix, Figure 14).
A		Cross-sectional area of pile base: m^2
u		Perimeter of the pile shaft: m
γ_{cf}	Similar to α	The service factor for soil on the surface of the shaft, depending on pile installation method $\gamma_{cf} = 0{\cdot}6$ (Table 5, SNiP) – this is similar to the adhesion factor α
f_i		The design resistance of ith layer of soil on the surface of the shaft on driven and rotary bored piles, kPa, taken from Table 2 (SNiP 2.02.03-85). Depends on type of soils, liquidity index I_L and average depth of soil stratum location (design chart is shown in the Appendix, Figure 13)
h_i		The thickness of ith layer of soil, contacting the pile shaft: m

Table 9. Description of terms in SNiP 2·02·03-85 'Pile foundations' design equation

The factored pile resistance should be taken based on the condition

37. $$N = F_d/\gamma_k$$

γ_k – factor of safety = 1·4 (see SNiP 2.02.03-85, item 3.10)

For standard buildings the typical partial factors on variable (V) and permanent loads (G) are 1·2. (SNiP 2.01.07-85* 'Loads and effects', SNiP (1985b)). For SNiP calculations Equation 5 needs to be completely re-written as Equation 38

5. $$Q_d = \beta_1 G + \beta_2 V = \left[\frac{\pi D\alpha\int_o^L (c_u/\beta_3)\mathrm{d}z}{\beta_5} + \frac{A_b N_c(c_u/\beta_4)}{\beta_6}\right]\Big/\beta_7$$

38. $$N = \beta_1 G + \beta_2 V = \left[\frac{\gamma_{cR} RA}{(1{\cdot}0/\gamma_c)} + \frac{u\gamma_{cf}\sum f_i h_i}{(1{\cdot}0/\gamma_c)}\right]\Big/\gamma_k$$

39. $$N = 1{\cdot}2G + 1{\cdot}2V = \left[\frac{\gamma_{cR} RA}{(1{\cdot}0/1{\cdot}0)} + \frac{u\gamma_{cf}\sum f_i h_i}{(1{\cdot}0/1{\cdot}0)}\right]\Big/1{\cdot}4$$

13.1 Design calculation

For a 15 m long pile (12 m into the clay) and 0·45 m diameter and taking an $I_L = 0{\cdot}2$ (limit of SNiP) the skin friction calculation is summarised as Table 10.

Using Table 7 from SNiP 2.02.03-85 (Figure 14, in the Appendix of the current paper) and a representative I_L of 0·1 at the pile toe depth of 12 m below top of bearing stratum (SNiP is not clear if depth is below ground level or top of bearing stratum) the base resistance is R = 1400 kPa.

40. $$N = 1{\cdot}2G + 1{\cdot}2V = \left[\frac{1{\cdot}0\times1400\times0{\cdot}45^2(\pi/4)}{(1{\cdot}0/1{\cdot}0)} + \frac{\pi\times0{\cdot}45\times456{\cdot}7}{(1{\cdot}0/1{\cdot}0)}\right]\Big/1{\cdot}4$$

Taking $V = 0{\cdot}25$G

41. $$N = 1{\cdot}2G + 1{\cdot}2(0{\cdot}25G) = (222{\cdot}7 + 645{\cdot}6)/1{\cdot}4$$
$$N = 1{\cdot}5G = (222{\cdot}7 + 645{\cdot}6)/1{\cdot}4$$

$G = 413{\cdot}5$ kN
$V = 103{\cdot}3$ kN
$Q_{work} = 516{\cdot}8$ kN

The equivalent FOS = (868·3/516·8) = 1·68. The calculated shaft

Material	Layer	Depth to mid-point: m	h_i: m	I_L	γ_{cf}	f_i: kPa	$\gamma_{cf} \times f_l \times h_i$: kPa m
Made ground	1	0·5	1	0·0	0·0	0·0	0·0
Made ground	2	1·5	1	0·0	0·0	0·0	0·0
Made ground	3	2·5	1	0·0	0·0	0·0	0·0
Stiff clay	4	3·5	1	0·2	0·6	49·5	29·7
Stiff clay	5	4·5	1	0·2	0·6	53·1	31·9
Stiff clay	6	5·5	1	0·2	0·6	56·2	33·7
Stiff clay	7	6·5	1	0·2	0·6	58·9	35·3
Stiff clay	8	7·5	1	0·2	0·6	61·3	36·8
Stiff clay	9	8·5	1	0·2	0·6	63·5	38·1
Stiff clay	10	9·5	1	0·2	0·6	65·5	39·3
Stiff clay	11	10·5	1	0·2	0·6	67·4	40·4
Stiff clay	12	11·5	1	0·2	0·6	69·1	41·5
Stiff clay	13	12·5	1	0·2	0·6	70·7	42·4
Stiff clay	14	13·5	1	0·2	0·6	72·3	43·4
Stiff clay	15	14·5	1	0·2	0·6	73·7	44·2
					$\sum$		456·7

Table 10. Example of SNiP 2.02.03-85 calculation

and base resistances from the 'αc_u' method (832·6 kN and 225·2 kN) gives a combined resistance of 1057·8 kN, which is not too dissimilar to the 868·3 kN from the SNiP calculation. The correlations implicit in SNiP seem to give capacities very similar to UK practice.

14. Summary of results

Table 11 summarises calculations for the 0·45 m diameter, 15 m long pile analysed throughout the paper, using the seven design methods. Figure 10 shows the calculated combined unfactored allowable loads for a 0·45 m diameter pile (Q_{work}) for the various design approaches with respect to pile lengths from 10 to 20 m. Figure 11 shows the same for a 0·9 m diameter pile. Figure 12 shows the global factor of safety for the 0·9 m pile. Most codes have a consistent factor of safety; the UK value drops slightly as the pile lengthens, as the base resistance is less significant and it is the base that has the higher partial factor. The DA2 approach (with the reduction for a bored pile) has an increasing factor of safety as the pile lengthens, as only the skin friction is reduced. In all other cases a single FOS value is used over the range of pile lengths studied. Coincidentally, the DA3 calculations and the AS2159 (Australian) calculations basically give the same results in terms of pile length and overall FOS. Therefore the lines on Figures 10–12 are virtually indistinguishable.

Code	G: kN	V: kN	Q_{work}: kN	Equivalent FOS	β factors used by the code for this design (non-unity)
Conventional design	305·4	76·4	381·8	3·0	β_7
AS2159-2009	349·8	87·5	437·3	2·42	β_1, β_2 and β_7
EC7-UK DA1-2	341·2	85·3	426·5	2·48	β_2, β_5, β_6 and β_7
EC7-Ireland DA2	318·6	79·6	398·2	2·66	β_1, β_2, β_5, β_6 and β_7
EC7-The Netherlands DA3	348·2	87·1	435·3	2·43	β_3, β_4, β_5 and β_6
AASHTO (USA)	248·0	62·0	310·0	3·85[a]	β_1, β_2, β_5, β_6 and β_7
SNiP (Russia)	413·5	103·3	516·8	1·68	β_1, β_2 and β_7

[a] 3·41 if compare capacity with shaft and base resistances calculated using 25th percentile through the undrained shear strength data. (see Figure 8)

Table 11. Summary calculations, 0·45 m diameter; 15 m long pile

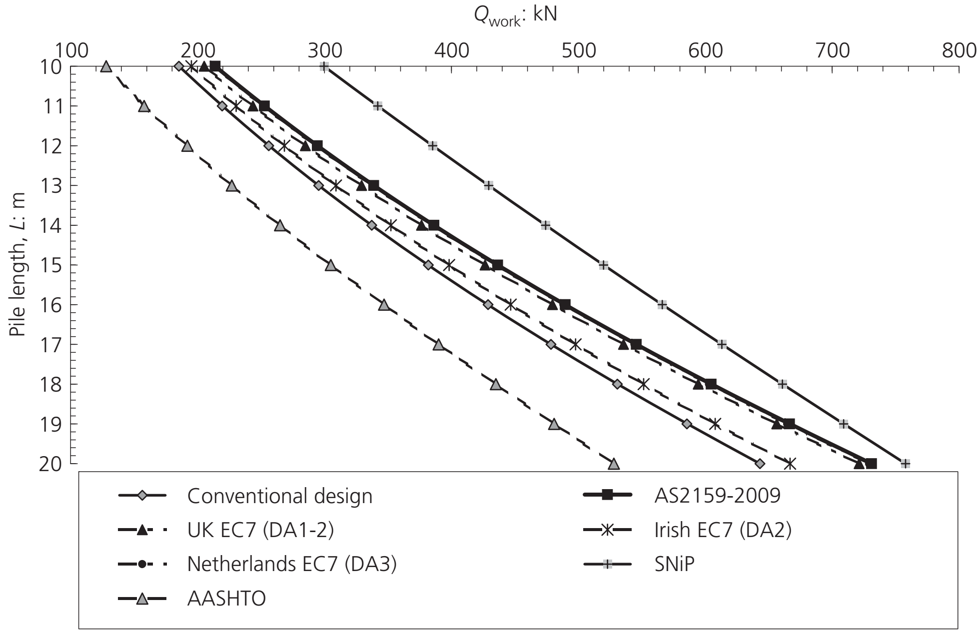

Figure 10. Unfactored working load plotted against pile length (0·45 m diameter pile)

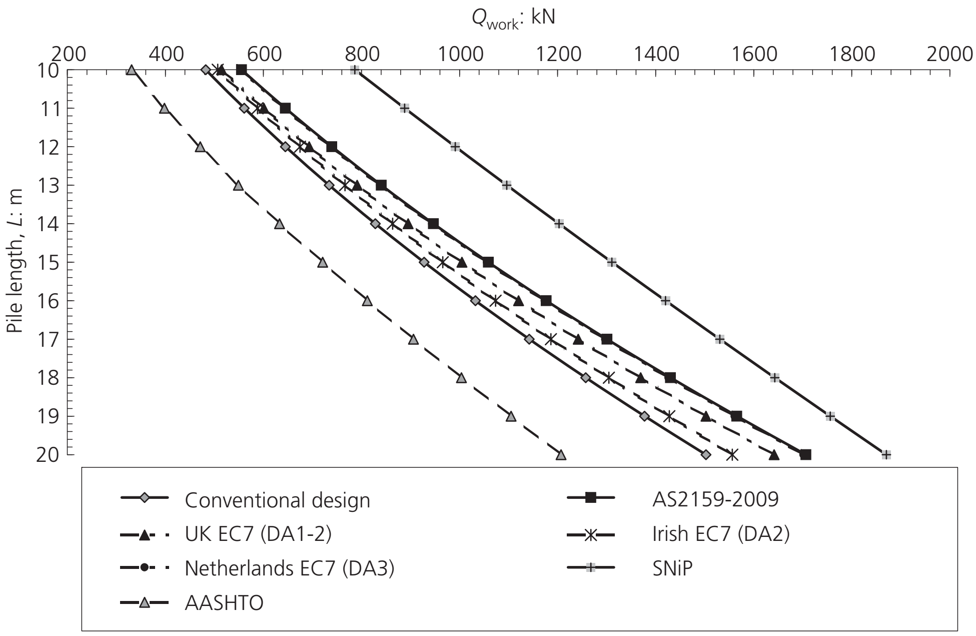

Figure 11. Unfactored working load plotted against pile length (0·9 m diameter pile)

15. Discussion and conclusion

The following observations are made based on the study described in the current paper.

(*a*) The UK (DA1), Netherlands (DA3) and AS2159 calculations give closely similar results (for this design example, using the α-method of calculation) with a global FOS of just under 2·5. The Irish DA2 approach gives a slightly higher FOS value. The difference occurs when AASHTO and SNiP are considered. AASHTO is a very conservative code as the factors and the loading and resistance are very high. AASHTO would be even more conservative if the design line

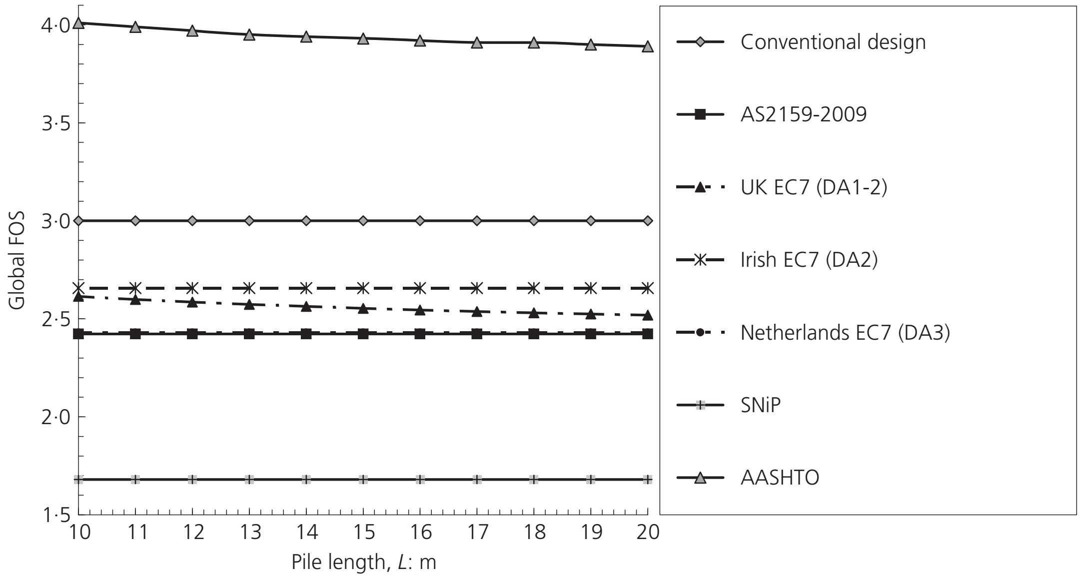

Figure 12. Global FOS plotted against pile length (0·9 m diameter pile)

(Equation 8a) was used instead of Equation 8c. The fact that AASHTO is mainly a bridge code could be why the variable loading factor of 1·75 is very high and why designs are very conservative. The SNiP calculations are significantly less conservative.

(*b*) Most codes have the flexibility of applying different factors to the shaft and base resistance. The base is generally factored higher as more uncertainty exists in the determination of what the pile is founded in and how much the base is disturbed by construction. The Australian and Russian codes use a single reduction factor applied to the combined resistances.

(*c*) The Australian code is unique in that the engineer has input into the factor of safety chosen by means of a simple risk analysis approach. This recognises that site conditions dictate the amount of uncertainty in the design to a certain extent and that a 'one-size-fits-all' approach can constrain engineering judgement which is crucial for good design. This could also be seen in the Eurocode context as embodied in the use of a 'cautious estimate', which is perhaps a more abstract concept that achieves a similar result.

(*d*) Direct comparison of the allowable working pile resistance, Q_{work}, for each code is obscured by the fact that different estimates of shear strength were used, especially in AASHTO and SNiP where triaxial data only and liquidity index correlations respectively are used to derive a c_u profile and f_i profile respectively.

(*e*) There is little guidance in any of the design codes on how to construct a design line for the shear strength profile. Some codes specify (or imply) the use of average soil parameters while Eurocode 7 (design by calculation) requires the use of a 'characteristic design line' which is a 'cautious estimate'. Code drafters could adopt a statistical approach (e.g. mean or 5th percentile); however, it is considered that this ignores the causes of ground variability. The use of a 'cautious estimate' or similar concept does allow the engineer a degree of flexibility in this respect. If the engineer accepts each data point as equally valid then a design line could be derived statistically. It does seem curious that partial factors can be assigned without knowledge of how conservatively engineers treat their soil data. If average soil values are to be used in design then higher partial factors are needed than if 5th percentile values are used. This is investigated further in the companion paper (Vardanega *et al.*, 2012).

(*f*) A complication when comparing different codes of practice is that permanent and variable loads are factored differently from code to code. For a fair comparison of codes, the factors on loads (actions) and resistances need to be brought together. The key to success is that there is a clear understanding between structural and geotechnical engineers as to who applies the partial factors on actions.

(*g*) For bored piles in London Clay 'α' of 0·5 is generally recommended. The AASHTO approach and the SNiP approach use similar values of shaft resistance. AASHTO has 'α' of 0·55 dropping gradually as estimates of c_u increase. In other words this code penalises a high c_u value.

(*h*) In SNiP the factor γ_{cf} can be interpreted as similar to the 'α' concept as it reduces the shear strength of the clay around a bored pile and relates to the method of installation. The use of liquidity index (I_L) is not without basis as relationships between I_L and c_u have been discussed (e.g. Muir Wood, 1983). Possibly, the use of I_L in SNiP 'works' because it indirectly measures values of c_u, which relate to shaft friction.

(*i*) An interesting feature of the AASHTO approach is that the SPT is not favoured for design; triaxial data are favoured. This is despite SPT data sometimes displaying less scatter than triaxial data (see Figures 4 and 5 and LDSA (2000)).

(*j*) The major reason SNiP appears unconservative is that the partial factor on resistance (1·4) and the partial factor on actions (1·2) are both relatively low. It is not known if the estimates of skin friction are conservative or not as the source of the data in SNiP Tables 2 and 7 (Figures 11 and 12 in this paper) is unclear. A comparison with αc_u values derived suggests that they are high at shallow depth and low at greater depth. Overall for the 12 m pile, there is little difference between the SNiP representative resistance and that derived from the 'α' method. It would be interesting to know performance statistics for piled foundation systems constructed under the SNiP framework.

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Appendix – SNiP design charts
For the shaft resistance of piles in clay, cubic equations of the form in Equation 42 were fitted to the data tables from SNiP. The regression coefficients are shown in Table 12 and the plotted functions in Figure 13.

42. $$f_i = a(I_L)^3 + b(I_L)^2 + c(I_L) + z$$

For the base resistance of piles in clay, linear equations of the form shown below were fitted to the data tables from SNiP. The regression coefficients are shown in Table 13 and the plotted functions in Figure 14.

43. $$R = A(I_L) + K$$

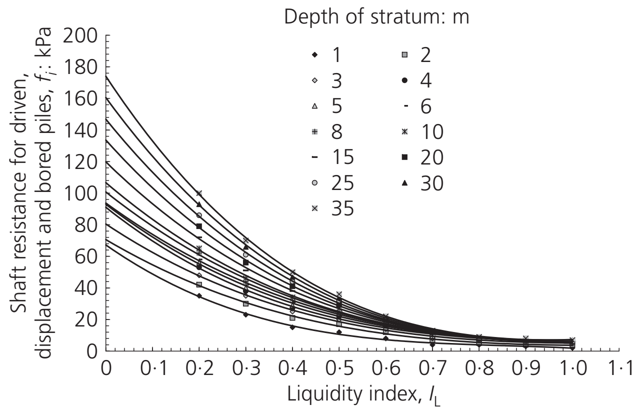

Figure 13. Graphical representation of Table 2 in SNiP 2.02.03-85

Stratum depth, d: m	d = 1	d = 2	d = 3	d = 4	d = 5	d = 6	d = 8	d = 10	d = 15	d = 20	d = 25	d = 30	d = 35
a	−90·1	−43·8	−48·0	−76·6	−58·9	−41·2	−47·1	−52·2	−61·4	−74·1	−79·1	−89·2	−107·7
b	230·7	151·3	168·5	225·8	198·2	168·2	186·7	203·7	236·0	275·6	303·5	341·1	388·4
c	−206·0	−174·5	−195·9	−235·1	−226·4	−214·7	−234·4	−252·2	−288·5	−329·2	−365·0	−405·1	−447·5
z	67·2	70·6	80·4	91·1	93·0	93·7	100·7	106·6	119·8	133·6	146·8	160·5	174·0

Table 12. Fitted coefficients (Table 2, SNiP)

Design resistance, *R*: kPa below the pile tip	*d* = 3	*d* = 5	*d* = 7	*d* = 10	*d* = 12	*d* = 15	*d* = 18	*d* = 20	*d* = 30	*d* = 40
A	−1054	−1107	−1196	−1250	−1446	−1679	−1911	−2107	−3300	−5000
K	845	875	1116	1325	1541	1811	2088	2304	3300	4500

Table 13. Fitted coefficients (Table 7, SNiP)

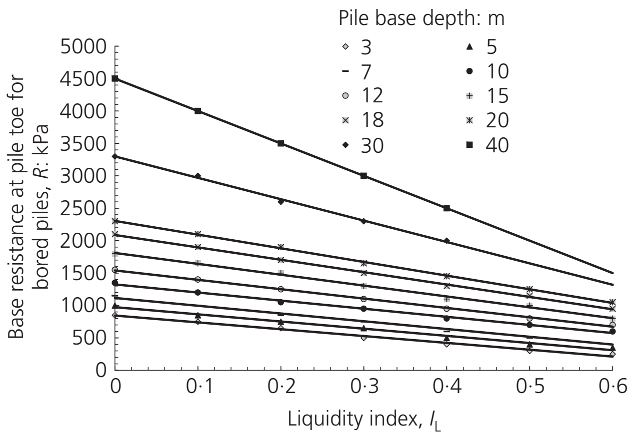

Figure 14. Graphical representation of Table 7 in SNiP 2.2.03-85

REFERENCES

AASHTO (American Association of State Highway and Transportation Officials) (2007) *LRFD Bridge Design Specifications*, 4th edn. AASHTO, Washington, DC, USA (with 2008 interim revisions).

Bond AJ and Simpson B (2010) Pile design to Eurocode 7 and the UK National Annex Part 2: UK National Annex. *Ground Engineering* **43(1)**: 28–31.

BSI (1986) BS 8004: 1986: Code of practice for foundations. BSI, Milton Keynes, UK.

BSI (2007) National Annex to Eurocode 7: BS EN 1997-1-2004: Geotechnical design – part 1: general rules. BSI, Milton Keynes, UK.

BSI (2010) Eurocode 7: BS EN 1997-1-2004: Geotechincal design – part 1: general rules. BSI, Milton Keynes, UK (incorporating corrigenda February 2009).

LDSA (London District Surveyors Association) (ed.) (2000) *Guidance Notes for the Design of Straight Shafted Bored Piles in London Clay*. LDSA Publications, Bromley, UK, Guidance Note 1.

Meyerhof GG (1976) Bearing capacity and settlement of pile foundations (11th Terzaghi Lecture). *Journal of the Geotechnical Engineering Division, ASCE* **102(3)**: 195–228.

Muir Wood D (1983) Index properties and critical state soil mechanics. In *Proceedings of the Symposium on Recent Developments in Laboratory and Field Tests and Analysis of Geotechnical Problems, Bangkok, 6–9 December 1983*. Balkema, Rotterdam, the Netherlands, pp. 301–309.

NEN (Netherlands Standardization Institute) (2007) Eurocode 7: National Annex to NEN-EN 1997-1 – Geotechnical design – Part 1: General rules. NEN, Delft, the Netherlands.

NSAI (National Standards Authority Ireland) (2005) National Annex Eurocode 7: I. S. EN 1997-1-2005: Geotechnical design – Part 1: General rules. NSAI, Dublin, Ireland.

Patel D (1992) Interpretation of results of pile tests in London Clay. In: *Piling: European Practice and Worldwide Trends* (Sands MJ (ed.)). Thomas Telford, London, UK, pp. 100–110.

Poulos HG (2004) An approach for assessing geotechnical reduction factors for pile design. In *Proceedings of the 9th Australia New Zealand Conference on Geomechanics Auckland, New Zealand* (New Zealand Geotechnical Society and Australian Geomechanics Society (eds)), vol. 1, pp. 109–115.

Reid A and Taylor J (2010) The misuse of SPTs in fine soils and the implications of Eurocode 7. *Ground Engineering* **43(7)**: 28–31.

Simpson B, Calabresi G, Sommer H and Wallays M (1980) Design parameters for stiff clays. In *Proceedings of the 7th European Conference on Soil Mechanics and Geotechnical Engineering, Brighton, Design Parameters in Geotechnical Engineering* (British Geotechnical Association (ed.)), vol. 4, pp. 91–125.

Simpson B, Morrison P, Yasuda S, Townsend B and Gazetas G (2009) State of the art report: Analysis and design. In *Proceedings of the 17th International Conference Soil Mechanics Geotechnical Engineering, Alexandria, Egypt, The Academia and Practice of Geotechnical Engineering, 5–9 October 2009* (Hamza M, Shahien M and El-Mossallamy Y (eds)). IOS Press, vol. 4, pp. 2873–2929.

Skempton AW (1959) Cast in-situ bored piles in London Clay. *Géotechnique* **9(4)**: 153–173.

SNiP (1985a) SNiP 2.02.03-85: Pile foundations. Federal Registry of National Building Codes and Standards of Russia, Moscow, Russia.

SNiP (1985b) SNiP 2.01.07-85: Loads and effects. Federal Registry of National Building Codes and Standards of Russia, Moscow, Russia.

Standards Australia (2002) AS/NZS1170.0-2002: Structural design actions – part 0: general principles. Standards Australia, Sydney, New South Wales, Australia.

Standards Australia (2009) AS2159-2009: Piling – design and installation. Standards Australia, Sydney, New South Wales, Australia.

Stroud MA (1974) The standard penetration test in sensitive clays and soft rocks. *Proceedings of the European Seminar on Penetration Testing, Stockholm, Sweden*, vol. 2:2, pp. 366–375.

Tomlinson MJ (1986) *Foundation Design and Construction*, 5th edn. Longman Group, Harlow, UK.

Vardanega PJ and Bolton MD (2011) Strength mobilization in clays and silts. *Canadian Geotechnical Journal* **48(10)**: 1485–1503.

Vardanega PJ, Williamson MG and Bolton MD (2012) Bored pile design in stiff clay II: mechanisms and uncertainty. *Proceedings of the Institution of Civil Engineers – Geotechnical Engineering* **165(4)**: 233–246.